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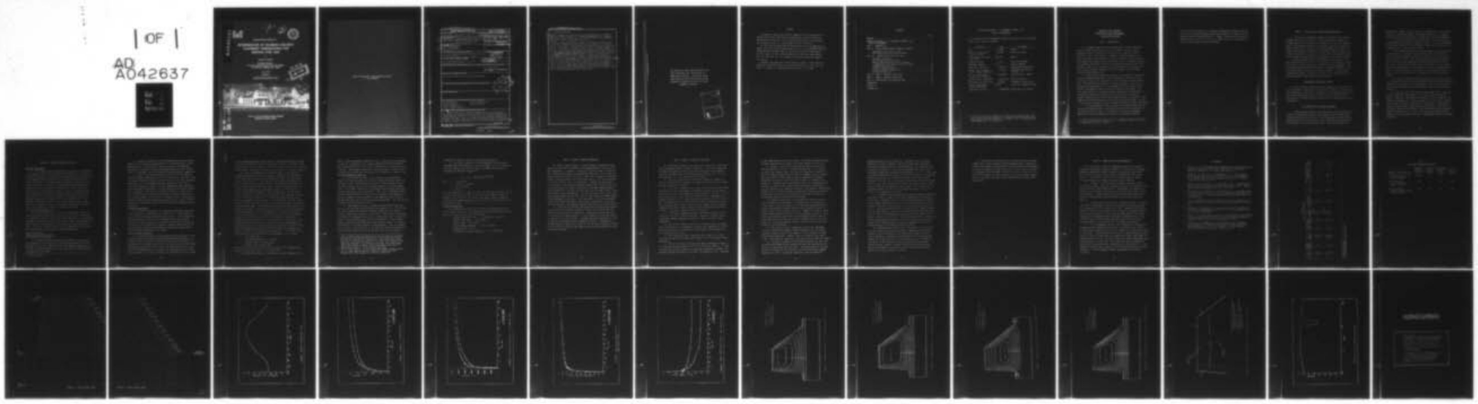

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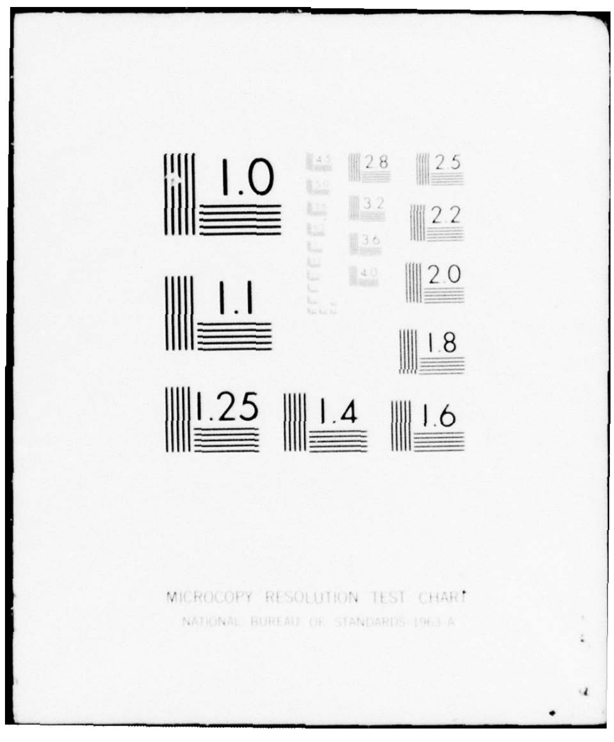
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DETERMINATION OF MAXIMUM CONCRETE PLACEMENT TEMPERATURES FOR MARTINS FORK DAM

by

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Finite element method computer programs were used to calculate the temperature rise, transient temperature change, and resulting thermal strains during simulated construction of Martins Fork Dam. The investigation was conducted in two phases. Phase I, involving temperature calculations only, was performed to provide temperature rise data upon which to base a revised maximum allowable concrete placement temperature that would reduce the potential for

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20. ABSTRACT (Continued).

thermal cracking. This information was needed before opening construction bids. As a result of Phase I, the recommended maximum allowable placement temperature was reduced from 85°F to 65°F for concrete with pozzolan replacement and to 60°F for concrete without pozzolan.

Phase II was conducted after aggregate sources were selected to verify the placement temperatures selected in Phase I. Thermal properties of the concrete used as input to the temperature calculation program were modified as dictated by the results of thermal diffusivity tests of aggregate samples and because of increased heat of hydration from the cement accepted for Martins Fork Dam. Computer temperature simulations used the revised thermal properties and new maximum placement temperatures. Two thermal stress/strain simulations used calculated temperature distributions to calculate thermal stresses and strains. Results showed that when exposed to normal ambient temperatures, surface tensile strains reached 34 percent of estimated tensile strain capacity. When exposed to an extreme mean ambient temperature drop of 30°F sustained for several days simulating a cold front, tensile strains in the dam reached 95 percent of strain capacity. It was concluded that the selected maximum allowable concrete placement temperatures were acceptable and that protection against rapid and sustained ambient temperature drops as from exposure to cold fronts would best be achieved by insulation of exposed concrete surfaces rather than by further reduction in placement temperature.

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PREFACE

A program to conduct mass concrete temperature rise simulations for Martins Fork Dam was authorized by DA Form 2544, Intra-Army Order for Reimbursable Services, No. 75-141, dated 29 April 1975, with attachment and inclosures, from U. S. Army Engineer District, Nashville.

The work reported herein was performed during the period May 1975 to August 1976 at the Concrete Laboratory of the U. S. Army Engineer Waterways Experiment Station (WES) under the direction of Messrs. Bryant Mather, Leonard Pepper, and Mrs. Katharine Mather, and under the supervision of Mr. B. R. Sullivan. Mr. A. A. Bombich was project leader and prepared this report.

Commanders and Directors of WES during the conduct of this study and preparation and publication of this report were COL G. H. Hilt, CE, and COL J. L. Cannon, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
pound force per square inch per minute	114.91267	pascal per second
pound (force) per square inch (psi)	6894.757	pascal
calorie per gram	4.184	Joule per kilogram
Fahrenheit degrees	5/9	Celsius degrees*
pounds per cubic yard	0.5932764	kilograms per cubic metre
Btu per hour · square foot · degree Fahrenheit	5.678263	watt per square metre · Kelvin
Btu · inch per hour · square inch · degree F	20.7688176	watt per metre · Kelvin
inch per degree Fahrenheit	0.014111111	metre per Kelvin
pound (mass) per cubic foot	16.01846	kilogram per cubic metre
Btu per pound (mass) · degree Fahrenheit	4186.8	Joule per kilogram · Kelvin
square foot per hour	0.0000258064	square metre per second

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings use: $K = C + 273.15$.

DETERMINATION OF MAXIMUM CONCRETE PLACEMENT TEMPERATURES FOR MARTINS FORK DAM

PART I: INTRODUCTION

1. A primary concern during and immediately following completion of mass concrete structures is control of thermal cracking. During construction heat is produced by the hydration of cement resulting in a temperature rise in the concrete. Subsequent thermal gradients occur due to cooling by external ambient temperature change. Concrete temperature change causes proportional volume change that if restrained internally or externally will produce thermal strains which if excessive will cause cracking. This study is concerned with determination of a maximum allowable concrete placement temperature that will reduce possible thermal cracking in Martins Fork Dam.

2. This study was initiated in May 1975 to determine if the maximum concrete placement temperature of 85°F* designated for Martins Fork Dam was excessive and if so to determine a revised maximum placement temperature. The results were needed within a month in order to meet the construction bid opening date. It was decided that the study would be conducted in two phases in order to meet the deadline requirements. The first would consist of conducting thermal simulations of construction to determine the temperature distribution in the structure considering several construction start dates and maximum placement temperatures. Peak temperatures would be obtained from these calculations and used as a guide in the decision regarding a change in maximum allowable placement temperature.

3. The second phase of the study was to be conducted after the contractor was determined and the quarries selected from which aggregate for the concrete would be taken. An evaluation of the selected aggregate would then be made and modifications made to the thermal properties

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* A table of factors for converting U. S. customary to metric (SI) units of measurement is given on page 4.

of the concrete if necessary. Additional temperature simulations would be run using the maximum placement temperature selected after Phase I and the modified concrete thermal properties. Thermal stress analysis simulations would be made at this time. The goal of Phase II would be verification of the selected maximum placement temperature.

PART II: FINITE ELEMENT METHOD COMPUTER PROGRAMS

4. Two each two-dimensional finite element method (FEM) computer programs were used in this study. The first program, developed by Dr. Edward Wilson of the University of California at Berkeley[1] and modified at the Waterways Experiment Station (WES), calculates temperatures within a mass concrete structure. A second program, written by R. S. Sandhu and associates at Berkeley[2] and modified at WES, calculates the thermal stresses and strains within the structure resulting from gravity and the thermal loads produced by the temperature calculation program.

5. Both programs use the same FEM model of the structure. The model subdivides the structure into a grid pattern in which the intersection points are called nodes and the enclosed areas are elements. Lift and material interfaces must correspond to an element boundary. A plot of the model used in this study is shown in Figure 1. Both programs incrementally simulate placement of concrete in lift thicknesses and at elapsed placement times as prescribed by the user.

Temperature Calculation Program

6. The temperature program calculates temperatures at each node in the FEM model. Temperature calculations are based upon concrete placement temperature, hydration heat generated, and the thermal properties of the concrete which govern heat flow within and loss or gain from the structure due to ambient conditions controlled by a surface heat transfer coefficient.

The Stress/Strain Calculation Program

7. This program calculates the displacements at each node and the strains and stresses developed in each element in the FEM model due to thermal and gravity loads. When creep is considered, stresses at each analysis time step are modified for stress relaxation with no strain for the interval up to the next analysis time. The creep parameters are

stored and the change in stress stored as residual stress to be included in the next time step analysis. When these stored values are applied during the next time step analysis, strains due to creep are eliminated.

8. Since creep removes those strains due to plastic deformation, the remaining strain should be completely elastic. In order to determine if the strains calculated are sufficient to cause cracking they must be compared against a crack threshold strain.

9. The cracking threshold used is the ultimate rapid-load tensile strain capacity.[3] Rapid load strain capacity tests are conducted at a rate of loading (40 psi/min) sufficiently rapid to not allow significant plastic strains to occur. Thus, elastic tensile strains calculated in the FEM analysis can be compared with tensile strain capacity for the age of the concrete in the element under consideration. If the tensile strain reaches 100 percent of strain capacity, it can be assumed that the onset of cracking has begun. Discussion of stress analysis results in Phase II will be in terms of percent tensile strain to tensile strain capacity.

10. The stress program simulates construction in the same manner as the temperature program for a given problem solution and uses the nodal temperatures calculated to determine thermal loads. The stress program requires time-dependent material properties for each unique material in the model.

11. The input value that instructs the program when to apply temperature changes as volume changes is the stress-free temperature. A value of stress-free temperature is determined for each element by the temperature program at 8 hr after placement and is the value of temperature at which an element is assumed to be stress-free. Subsequent temperature changes produce volume changes proportional to the coefficient of thermal expansion of an element. When differential volume changes are produced, stresses result. Stresses and strains calculated are also functions of initial external forces or displacements applied as boundary conditions.

PART III: COMPUTER MODEL AND INPUT DATA

FEM model development

11. The computer model developed for this study is representative of the general cross-section configuration of monoliths M-3 through M-9 for Martins Fork Dam. The FEM model was obtained using computer graphics and the WES FEM Pre-Processor System. The model was developed up to elevation 1305 ft. It was assumed that this lower portion of the structure would be most affected by the maximum placement temperature and would be the area most likely to be subjected to thermal cracking. A computer plot of the FEM model is shown in Figure 1. Included is the lift layout and node and element numbering scheme. The upstream and downstream faces of the model to a depth of two elements consist of exterior concrete with a cement factor of 4.0 bags per cu yd.* The remainder of the model consists of an interior concrete with a cement factor of 3.5 bags per cu yd. The foundation is modeled to a depth of 35 ft. All galleries were ignored to expedite the study.

Lift height and placement rate

12. The lift heights specified in the construction schedule provided by Nashville District were 3 each, 2-1/2-ft lifts to be placed on the foundation, followed by 5-ft lifts for the remainder of the structure provided the time between lifts is less than 15 days. The placement rate agreed upon was 4 days between 2-1/2-ft lifts and 6 days between 5-ft lifts. This is the probable fastest rate of construction, and the rate that would probably produce maximum internal temperatures.

Ambient temperatures, concrete placement temperatures, and construction start dates

13. An annual ambient temperature curve (Figure 2) was derived from information which represented the mean monthly temperature for the geographic area of the construction site. The ambient temperature for any day is the mean temperature for that date; thus, the daily variation in temperature is not used.

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* A "bag" = 94 lb.

14. Placement temperatures in the simulations were taken from the ambient curve for the dates corresponding to the placement date for any particular lift. The exception to this procedure was in simulations where ambient temperature exceeded prescribed maximum placement temperature. Then placement was at the prescribed maximum placement temperature.

15. Table 1 lists the construction start dates and the maximum concrete placement temperature for all runs conducted in this study. Runs 1-3 compared construction start dates of 1 April, 1 June, and 1 September, respectively, at a 72°F maximum placement temperature. Run 4 with a 1 June start showed the effects of an 80°F maximum placement temperature. This corresponds to placement of concrete exclusively at ambient temperature in midsummer. Runs 5-8 were conducted in Phase II of the study. Run 5, duplicating run 2 with modified concrete thermal properties had a 72°F placement limit. Runs 6-8 had 65°F placement limits. This was the maximum allowable placement temperature agreed upon as a result of Phase I simulations. Runs 6-8 were made to verify the selected maximum placement temperature.

Foundation temperatures

16. In order to approximate the foundation temperature the bottom of the foundation in the model was set equal to 55°F. This temperature represents the mean annual temperature for the site. It was assumed that the temperature at the bottom of the foundation will remain constant throughout the year. Prior to the beginning of construction simulation, the surface of the foundation was exposed to ambient temperature and the simulation began equilibrating foundation temperatures between ambient and the constant temperature at the bottom.

Concrete thermal properties

17. Two concrete mixtures were specified for the structure. These included exterior and interior mixtures having cementitious material contents of 4.0 and 3.5 bags per cu yd with 25 and 35 percent pozzolan replacement by solid volume, respectively. No tests to determine the thermal or mechanical properties of the concrete were authorized for this study. Thus, this information had to be obtained from existing data for similar concrete mixtures. The concrete thermal data used were from tests at WES

for the Tennessee-Tombigbee thermal study.[4] Slight modifications were made to account for different cementitious material contents. For Phase I, runs 1-4, a conductivity of 0.1054 Btu·in./hr·in.^{2}·F and specific heat of 0.22 Btu/lb·F were used. Adiabatic temperature rise curves used are shown in Figure 3.

18. Phase II simulations were conducted after the Martins Fork aggregate quarries were selected. Samples of aggregates from two quarries were obtained and tested for thermal diffusivity (CRD-C 36)[5] and modulus of elasticity (CRD-C 19).[5] The results of these tests are displayed in Table 2 along with the same properties for the aggregate used in the concrete from which Phase II data were taken. Since the thermal diffusivity of Martins Fork aggregate was substantially lower, the diffusivity of concrete containing this aggregate would also be lower. Thermal conductivity is the input property for heat flow used by the temperature program and is a function of diffusivity. A revised concrete diffusivity with Martins Fork aggregate was necessary. In the absence of an actual thermal diffusivity test of concrete containing Martins Fork aggregate an analytical determination was made. The assumption was made that concrete diffusivity is equal to the sum of the volumetric diffusivities of the coarse aggregate and mortar components. Since the aggregate and concrete mixture diffusivities were known for the existing Tenn-Tom concrete data, the unknown diffusivity of the mortar was found by the volumetric relationship of aggregate and mortar diffusivity to the diffusivity of the mixture. Once the diffusivity of the paste was determined, then the new aggregate diffusivity was substituted for the old and the new mixture diffusivity found. Using this procedure the concrete thermal diffusivity was reduced from 0.0375 to 0.0310 ft^2/hr with Martins Fork aggregate. The revised concrete conductivity was then calculated by the relationship $k = \alpha\rho c$, where

k = new conductivity

α = new thermal diffusivity of concrete

ρ = concrete density (assumed same)

c = specific heat (assumed same).

The resulting conductivity was 0.0875 Btu·in./hr·in.^{2}·F. Specific heat used was the same as in Phase I, or 0.22 Btu/lb·F.

19. Another FEM temperature program input property changed for the

Phase II runs was adiabatic temperature rise. This was due to an increase in the heat of hydration of the cement accepted* for Martins Fork Dam. The increase was from 70 to 73 cal/g at 7 days age or an increase of about 4 percent. The adiabatic temperature rise curves shown in Figure 3 were increased by the additional percentage of heat production in Phase II runs.

Concrete mechanical properties

20. At the time that Phase II was conducted, during February-August 1976, the Bay Springs Lock and Dam thermal study** was also being conducted at WES. Material properties such as Young's modulus, creep, and strain capacity versus time were being obtained for the Bay Springs study. The concrete mixtures for Bay Springs and Martins Fork were basically the same except for the aggregate. Thus, once the aggregate source was known for Martins Fork concrete, it was possible as a minimum verification to compare the elastic modulus of the aggregate used in the Bay Springs study with that of the Martins Fork Dam aggregate. As noted earlier, comparative values are found in Table 2.

21. The comparative values of modulus between Martins Fork aggregate (8×10^6 psi) and the Bay Springs study aggregate (11×10^6 psi) were not thought substantial enough to attempt a modification of concrete material properties for Phase II of this study. Since in fact the moduli for Martins Fork aggregate were lower, it was believed that the calculated thermal strain evaluations may tend to be conservative. This is based upon the apparent relationship that ultimate strain capacity is inversely proportional to the elastic modulus of the aggregate. Thus, any comparison of strain to strain capacity in the simulation should be conservative. The decision was made to use the Bay Springs study concrete material

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* Type II cement with heat-of-hydration option invoked was specified. This requires the heat-of-hydration to be not greater than 70 and 80 cal/g at 7 and 28 days, respectively. Some cement was accepted that had a 7-day heat of hydration of 73 cal/g. Accordingly, it was requested that Phase II be conducted using this value even though only a small amount of cement having a determined 7-day heat of hydration in excess of 70 cal/g was actually used.

** Letter report from U. S. Army Engineer Waterways Experiment Station to U. S. Army Engineer District, Nashville, subject and date: Thermal Study - Bay Springs Lock and Dam, 20 May 1976.

properties for Phase II of this study without modification.

22. The curves representing the time-dependent Young's modulus and rapid-load strain-capacity are shown in Figures 4 and 5, respectively. The form in which creep data are input to the FEM stress program is represented by McHenry's equation[6, 7]

$$\varepsilon_c(\sigma,\ t,\ T) = \sigma \sum_{i=1}^{N} A_i(T)\left(1-e^{-m_i(t-T)}\right)$$

where ε_c = creep strain

t = time after placement

T = age at loading

σ = stress.

N = 2 was found to give a satisfactory fit with experimental data. Values of creep coefficients A_1 and A_2 versus time are given in Figure 6. Values of $m_1 = 0.36$ and $m_2 = 0.0145$ were used. Constant values for coefficient of thermal expansion to 5.64×10^{-6} in.·F and Poisson's ratio of 0.174 were used.

Foundation properties

23. Foundation properties were derived from information provided by the Nashville District. The following thermal and mechanical properties were used as input values to the FEM programs.

thermal conductivity (wet) - 0.0925 Btu·in./hr·in.^{2}·F

specific heat (wet) - 0.35 Btu/lb·F

density (wet) - 166 lb/ft^3

elastic modulus (maximum) - 0.756×10^6 psi

Poisson's ratio - 0.38

coefficient of thermal expansion - 5.5×10^{-6} in./F

PART IV: PHASE I COMPUTER SIMULATIONS

24. Runs 1-4 made in Phase I of this study were conducted to help provide the basis for a revision of the maximum allowable concrete placement temperature. A summary of these runs is found in Table 1. Runs 1-3 were conducted to determine the temperature distribution in the structure after construction starts of 1 April, 1 June, and 1 September, respectively. Figures 7-10 are isotherm plots of the structure at 5 days after placement of the 14th lift for runs 1-4, respectively. Figures 7-9 clearly show the influence of the season during which construction took place since all concrete was placed at ambient temperature except where ambient temperature exceeded 72°F. The lower lifts of run 1 and the higher lifts of run 3 were placed at lower temperatures. All lifts of run 2 with a 1 June start were placed between 70°F and 72°F. Run 2 clearly shows much higher temperatures throughout the structure. Run 4 also with a 1 June start was made to show the maximum temperatures that should be achieved if no placement controls were made. The maximum placement temperature of any lift was 80°F.

25. The difference between peak temperature and mean annual temperature of 55°F or the eventual temperature drop of the structure was 49°F and 53°F for runs 2 and 4, respectively. Temperature drop has been used as a measure of cracking potential. It was thought that this drop should not exceed 45°F for this project. A maximum placement temperature of 70°F or lower was recommended by WES to Nashville District. Consequently, a maximum placement temperature of 60°F for concrete without pozzolan replacement and 65°F with pozzolan replacement was specified by Nashville District for Martins Fork Dam.

PART V: PHASE II COMPUTER SIMULATIONS

26. The primary purpose of this part of the study was to verify the selected maximum allowable placement temperature. This was accomplished by conducting temperature and thermal stress simulations using updated input data and applying the new maximum placement temperature. All runs were conducted with 1 June construction start dates.

27. Prior to conducting the verification simulation several temperature simulations were conducted to evaluate the effects of the thermal properties changes noted earlier.

28. Run 5 was the same as run 2 except that both the reduced thermal conductivity and revised adiabatic temperature rise were used to reflect those actually used. Peak temperature increased 2.4°F to 106.7°F. Run 6 was the same as run 5 except that the maximum placement temperature was reduced from 72°F to 65°F. Peak temperature was reduced by 3.9°F to 102.8°F. Run 7 was conducted to see the effect of the thermal conductivity change. This run was the same as run 6 except that the original concrete thermal conductivity was used. Results showed that peak temperature was reduced by 0.8°F. A review of temperature runs 5-7 follows.

29. A 7°F drop in maximum placement temperature from 72°F to 65°F produces a 3.9°F drop in peak temperature. Therefore a 1-degree change in maximum placement temperature produces a change of 0.55°F in peak temperature.

30. The revised thermal conductivity (18 percent lower) resulting from tests of Martins Fork aggregate increased peak temperature by 0.8°F. This is primarily due to a reduction in the amount of heat moving out of the structure between lifts.

31. The increase in heat of hydration limit from 70 to 73 cal/g at 7 days for Martins Fork cement will result in a 1.6°F increase in peak temperature.

32. The total effect of revising the thermal properties in Phase II is an increase in peak temperature of 2.4°F over that of Phase I. In order to equate the effects to produce the same peak temperature as Phase I the maximum placement temperature would need to be reduced by 4.5°F. Therefore

if peak temperature was the only criteria, the selected maximum allowable placement temperature would need to be further reduced by nearly 5°F.

33. However, this is not necessarily the case since the actual strains produced as a result of thermal gradients are the final guide for early time considerations. Run 6 was the temperature simulation in which the updated thermal properties and 65°F maximum placement temperature specified for concrete with pozzolan replacement were applied. Temperature histories of run 6 were used as input to a thermal stress simulation. Maximum tensile strains were found to be represented by the principal strains and were oriented along the exposed horizontal and vertical surfaces of the simulated structure at all stages of construction. Maximum ratios of tensile strains to tensile strain capacity occurred on the surfaces of each new lift beginning 2-3 days after placement and reached peak values as the next lift was placed.

34. Figure 11 depicts the areas in which the tensile strain to strain capacity ratio was greatest. Although the entire exposed horizontal surface of all lifts above lift 4 eventually experienced tensile strains, the maximum horizontal tensile strains reached 34 percent of strain capacity in the vicinity of point A. Maximum vertically oriented tensile strains reached 28 percent of strain capacity at point B. These values were reached on upper lifts of the simulated structure and the areas denoted were typically the points of maximum strains on all lifts except 1-4. The actual maximum values of tensile strain corrected for creep were 18-22 millionths.

35. The results of run 6 simulations indicate that midsummer construction with 65°F maximum placement temperature will not produce excessive thermal strains in late summer or fall if exposed to normal daily ambient temperature. However, this is a condition that rarely occurs. Cold fronts or temperatures unseasonably below normal are common.

36. Therefore, although not specifically requested in this study one additional simulation was conducted in which the structure was subjected to a simulated severe cold front. Run 8 duplicated run 6 except for the cold front that was simulated by a drop in mean daily ambient temperature of 30°F. The ambient temperature curve including the

temperature drop is shown in Figure 12. Although a drop in ambient temperature of 30°F is common, a 30°F drop in daily mean temperature sustained for several days is a severe deviation. The temperature drop occurred over a period of 2 days and was sustained for 4 days before returning to normal and coincided with placement of lift 11. In this case all freshly placed concrete surfaces of lift 11 are subjected to the temperature drop as well as the vertical surfaces of the older concrete. Exposure of the top lift to this ambient condition may very well be worse if the concrete were several days old before the condition began. However, the onset of the ambient drop was selected to fall within the lift placement schedule used in run 6 for comparison purposes; thus, only 6 days of lift exposure were possible.

37. A thermal stress simulation was conducted using the temperature histories of run 8. Tensile strains were compared with strain capacity as for run 6. Horizontal tensile strains reached 95 percent of tensile strain capacity along the top surface of lift 11 while vertical strains reached 70 percent of strain capacity on the lower vertical surfaces of lift 11. These values were reached 1 day after the 30°F ambient temperature drop was achieved. Tensile strains occurred along the vertical surfaces of older lifts varying from 67 percent of strain capacity on the surface of lift 10 to 5 percent of strain capacity on lift 2. Horizontal tensile strains were also observed in lifts 1-3 extending from the upstream to the downstream faces and reached a maximum of 15 percent of strain capacity at the midpoint between the faces.

38. Maximum tensile strains used in determination of the strain to strain capacity ratios were as follows. Maximum vertical tensile strains of 41-43 millionths occurred along the vertical surfaces of lifts 8 through 10. Horizontal tensile strains of 61 millionths were reached on the top of lift 11 in the vicinity of point A of Figure 11. The results of this simulation indicate that thermal cracking was narrowly averted. However, changes in any one of several properties used in this simulation could cause strain to exceed strain capacity because of the severe exposure condition.

39. The scope of this study did not include evaluation of other thermal strain control measures besides maximum placement temperature. It should be stressed that insulation of exposed surfaces would eliminate virtually all dangerous thermal strains due to severe ambient exposure. It is suggested that the cement contents of 3.5 and 4.0 bags per cu yd used in this study be reduced if possible, since the adiabatic temperature rise associated with such mixtures appears high for gravity dam construction.

PART VI: CONCLUSIONS AND RECOMMENDATIONS

40. As a result of the peak temperatures calculated from FEM temperature simulations conducted in Phase I of this study, the maximum placement temperatures allowed in Martins Fork Dam were reduced from 85°F to 65°F for concrete with pozzolan replacement and 60°F without pozzolan. Construction beginning in early summer produced temperature distributions most likely to produce thermal cracking.

41. In Phase II of this study two changes in concrete thermal properties were made. After the aggregate was selected for Martins Fork, it was tested for thermal diffusivity. Consequently, the thermal conductivity of the concrete was reduced by 18 percent with respect to that used in Phase I. Cement accepted for Martins Fork produced more than 4 percent additional heat than originally specified and simulated in Phase I.

42. A thermal stress run was made to simulate exposure to a severe cold front. The simulated cold front represented a 2-day drop in mean daily temperature of 30°F sustained for 4 days. Maximum tensile strain compared to tensile strain capacity was 95 percent on exposed horizontal surfaces and 70 percent on the upstream and downstream vertical faces. Lowering the 65°F maximum allowable placement temperature could not be justified based on these results especially since the input concrete properties were not derived from tests of Martins Fork concrete.

43. It is recommended, however, that in addition to observing the maximum placement temperatures specified, consideration also be given to insulating exposed surfaces to protect against the thermal shock effects of cold fronts. Insulation would also assist in reducing thermal strains that occur when late summer and fall temperatures remain below normal for extended periods of time. The rate at which the surface temperature drops during exposure to a cold front can be reduced by 60 to 80 percent[8] by application of insulation with a conductance of 0.5 $Btu/hr \cdot ft^2 \cdot F$. If so insulated, thermal strains observed in run 8 would be reduced by at least 30 percent. The 65°F maximum placement temperature should then be acceptable.

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Table 1

Computer Simulation Summary

Run No.	Simulated Start Date	Maximum Placement Temperature, Degrees F	Concrete Conductivity, Btu/lb·F	28-Day Concrete Adiabatic Temperature Rise, Degrees F*		Peak Concrete Temperature, Degrees F	ΔT Peak Temperature-Mean Ambient (55°F) Degrees F	Stress Calculation	Tensile Strain/Strain Capacity, Percent, Maximum			
				Exterior	Interior				Horizontal	Element No.**	Vertical	Element No.**
						Phase I						
1	1 Apr	72	0.105	52.1	46.1	99.4	44.4	No	--	--	--	--
2	1 Jun	72	0.105	52.1	46.1	104.4	49.4	No	--	--	--	--
3	1 Sep	72	0.105	52.1	46.1	94.5	39.5	No	--	--	--	--
4	1 Jun	80	0.105	52.1	46.1	107.7	52.7	No	--	--	--	--
						Phase II						
5	1 Jun	72	0.088	54.3	48.1	106.7	51.7	No	--	--	--	--
6	1 Jun	65	0.088	54.3	48.1	102.8	47.8	Yes	34	384/385	28	363/375
7	1 Jun	65	0.105	54.3	48.1	102.0	47.0	No	--	--	--	--
8	1 Jun	65	0.088	54.3	48.1	102.8	47.8	Yes	95†	308	70†	285/297

* Used in the calculation.
** Position of element numbers can be found in Figure 1.
† Occurred during exposure to simulated cold front.

Table 2

Comparative Aggregate Properties

Aggregate Project/Source	Thermal Diffusivity ft^2/hr	Young's Modulus 10^6 psi	Compressive Strength 10^3 psi	Poisson's Ratio
Bay Springs thermal study Calera, Alabama	0.053	11.0	20.0	--
Martins Fork Dam Southeastern Quarry Ewing, Virginia	0.031	7.5	15.7	0.20
Martins Fork Dam Kentucky-Virginia Quarry Ewing, Virginia	0.030	8.6	21.0	0.25

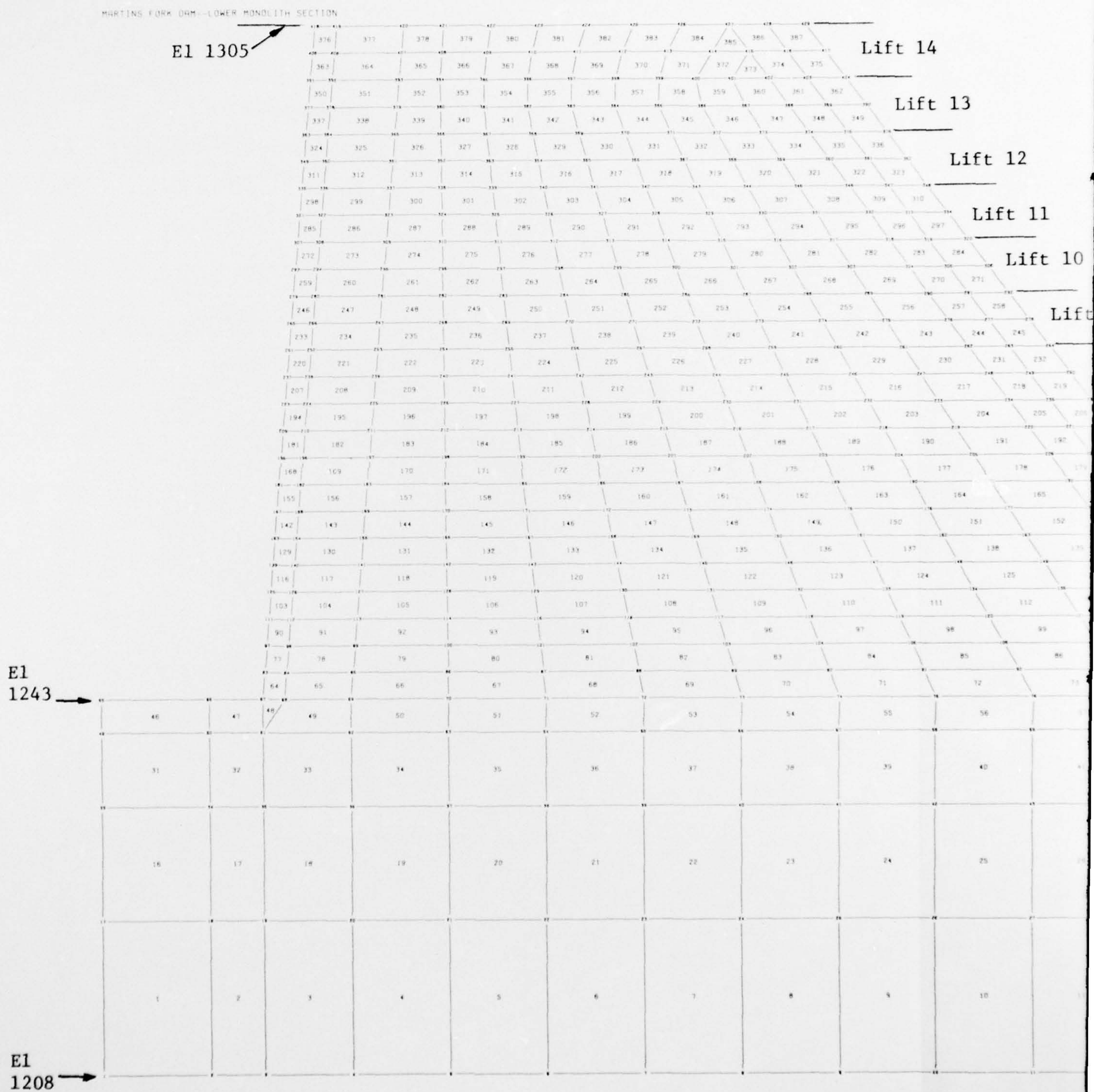


Figure 1. Finite element model

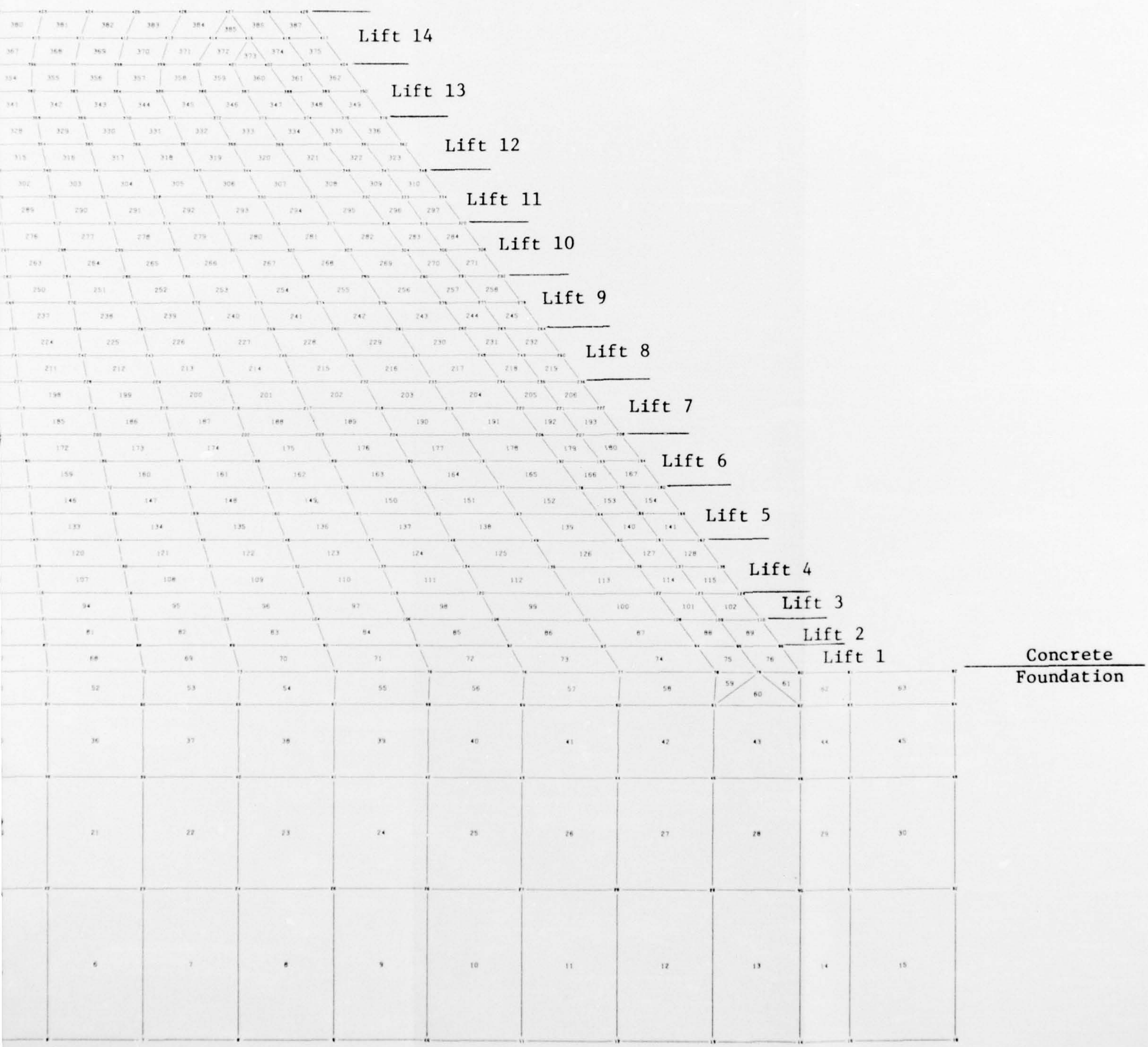


Figure 1. Finite element model

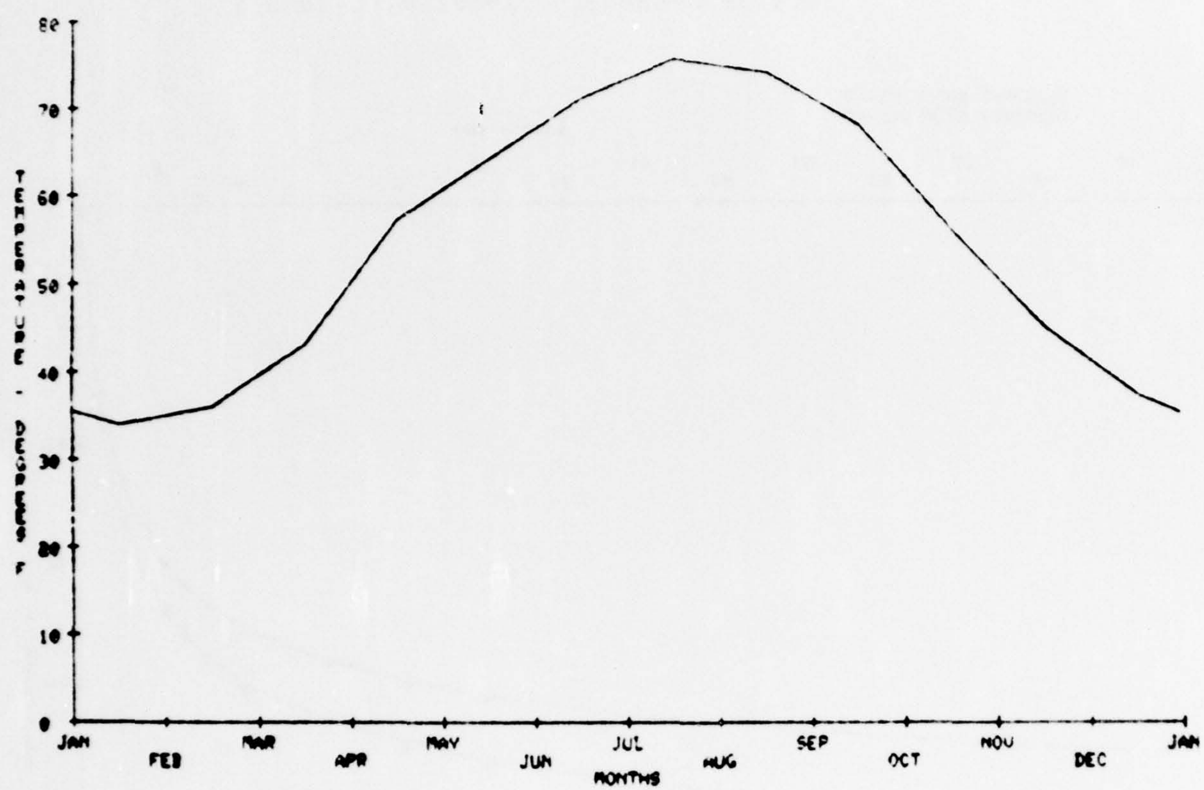


FIGURE 2 AMBIENT TEMPERATURE CURVE

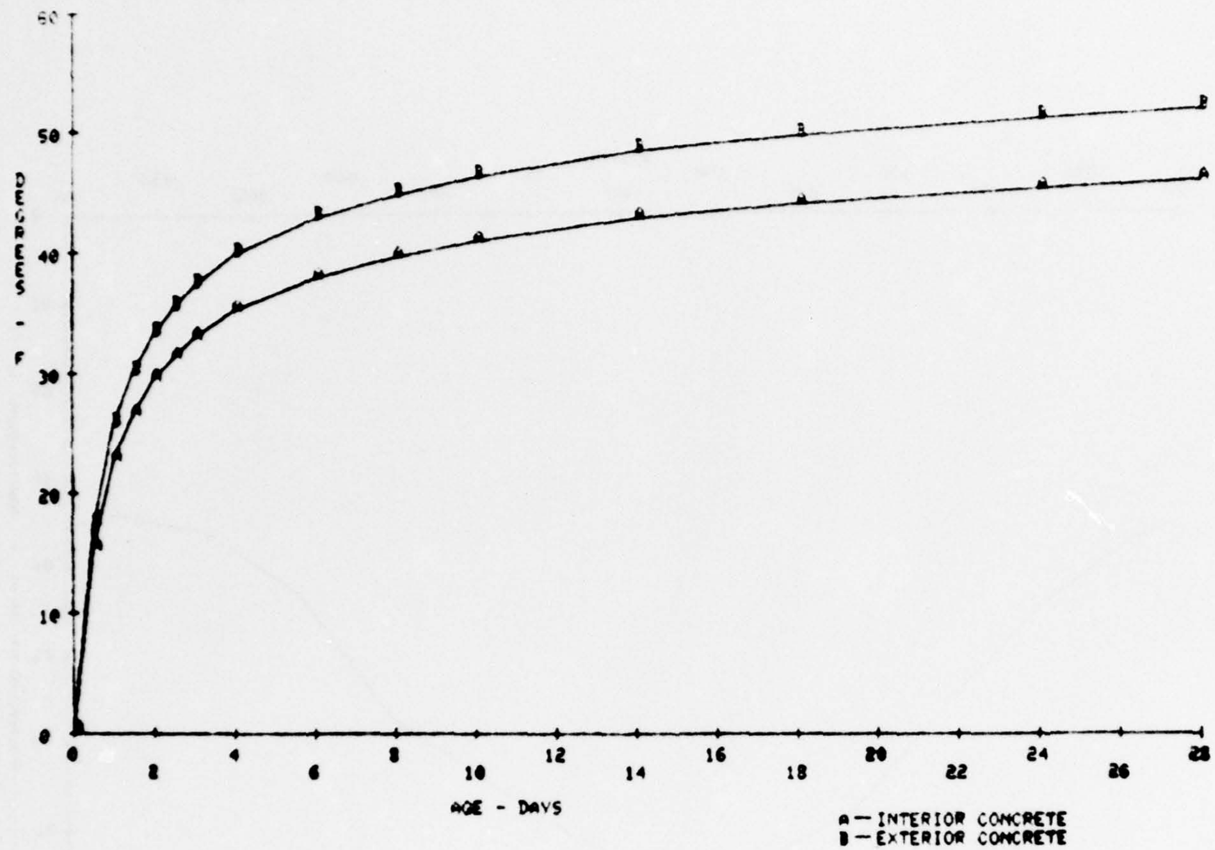


FIGURE 3 ADIABATIC TEMPERATURE RISE

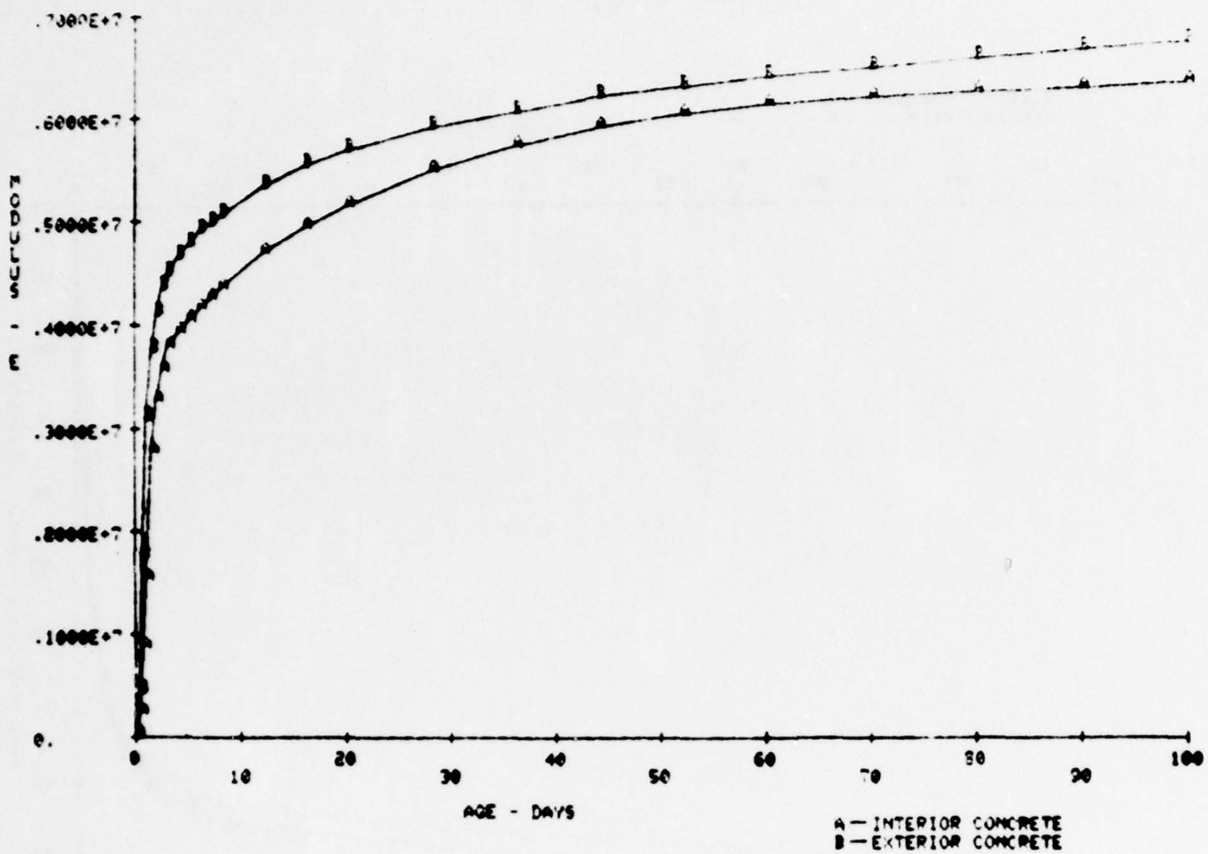


FIGURE 4 MODULUS OF ELASTICITY - BAY SPRINGS TESTS

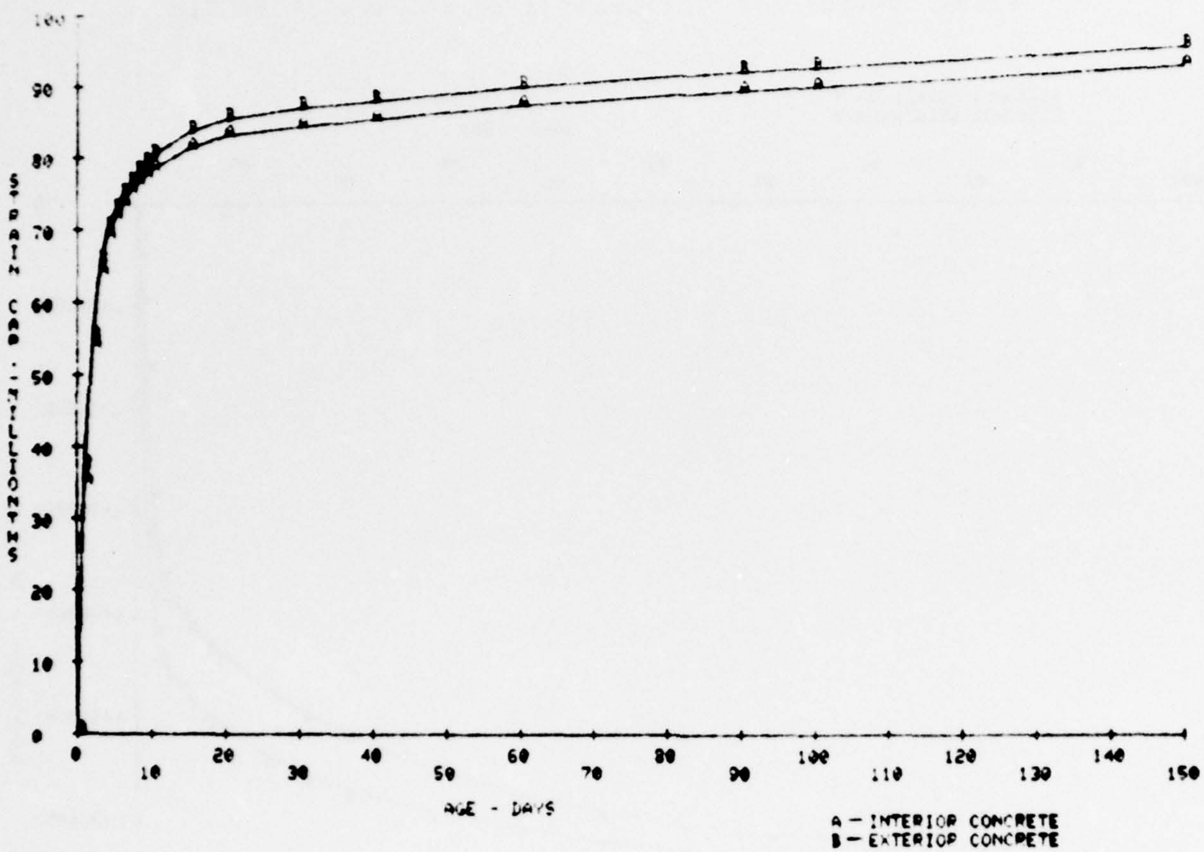


FIGURE 5 RAPID-LOAD STRAIN CAPACITY
-BAY SPRINGS TESTS

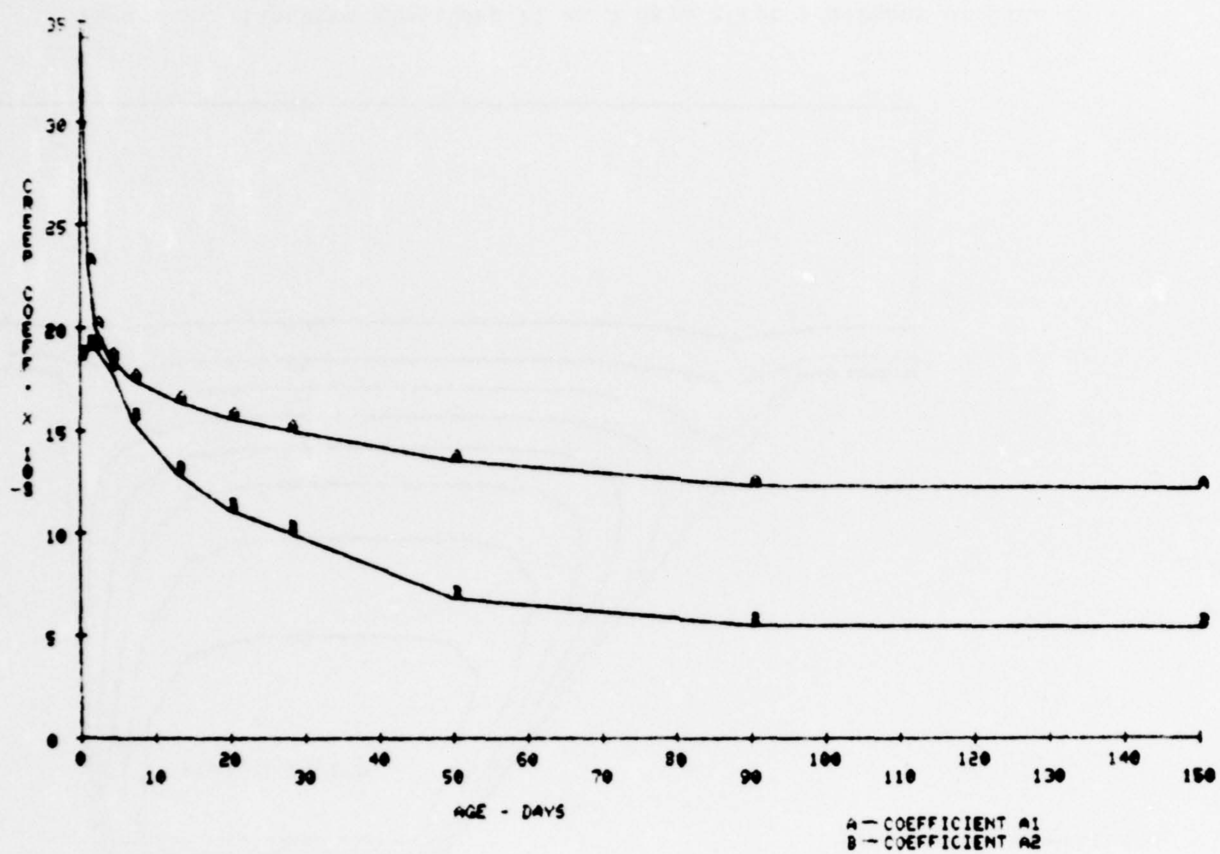


FIGURE 6 CREEP COEFFICIENT CURVES - BAY SPRINGS TESTS

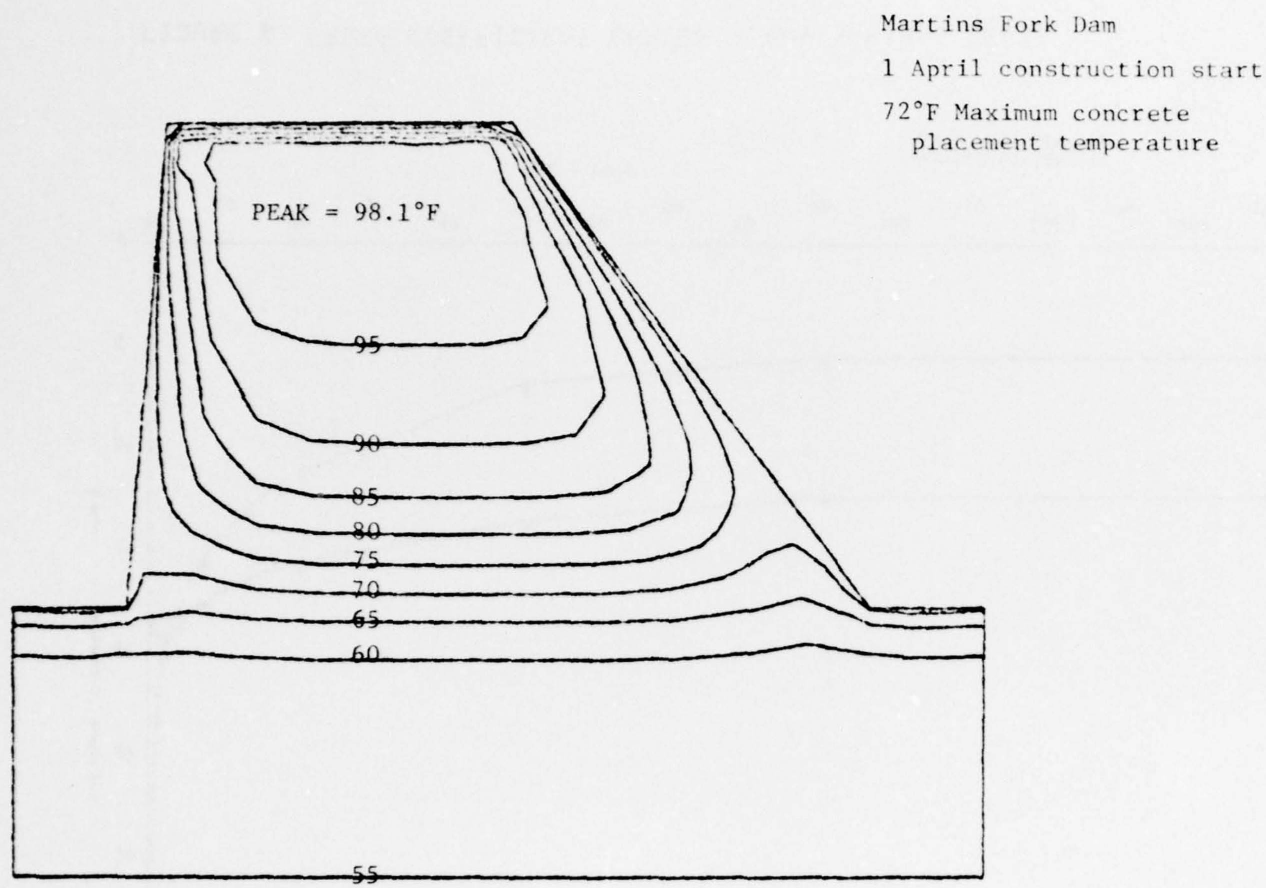


Figure 7. Isotherm plot (deg F) at 5 days after placement of Lift 14

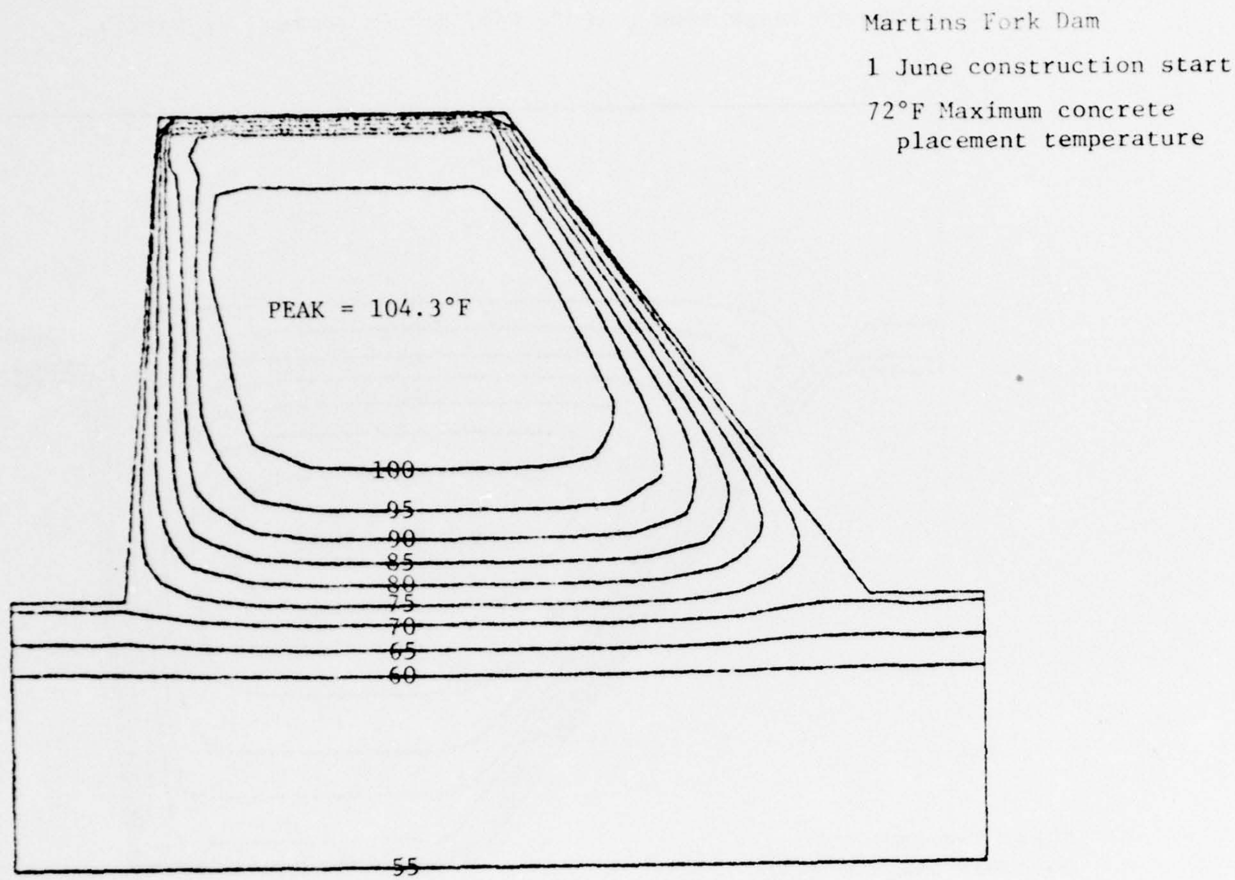


Figure 8. Isotherm plot (deg F) at 5 days after placement of Lift 14

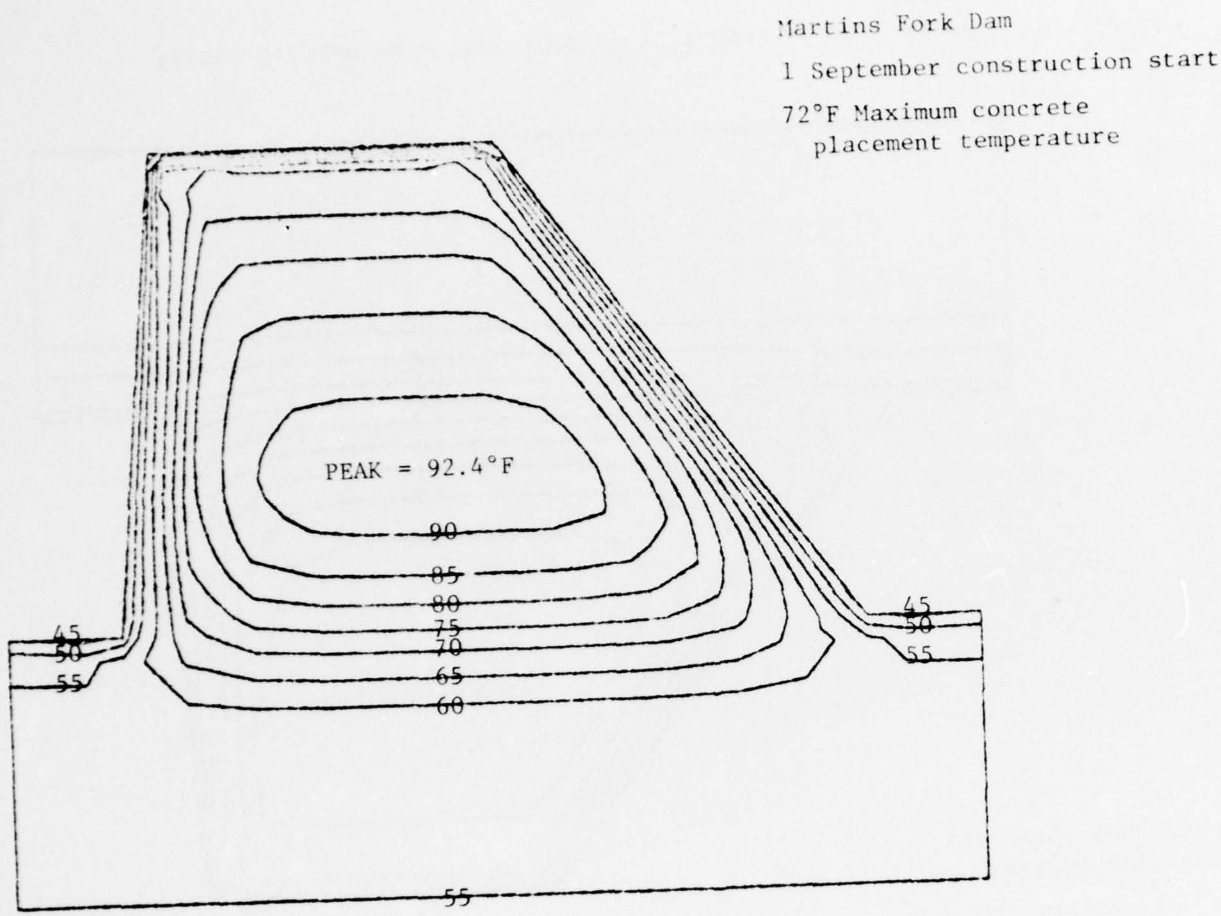


Figure 9. Isotherm plot (deg F) at 5 days after placement of Lift 14

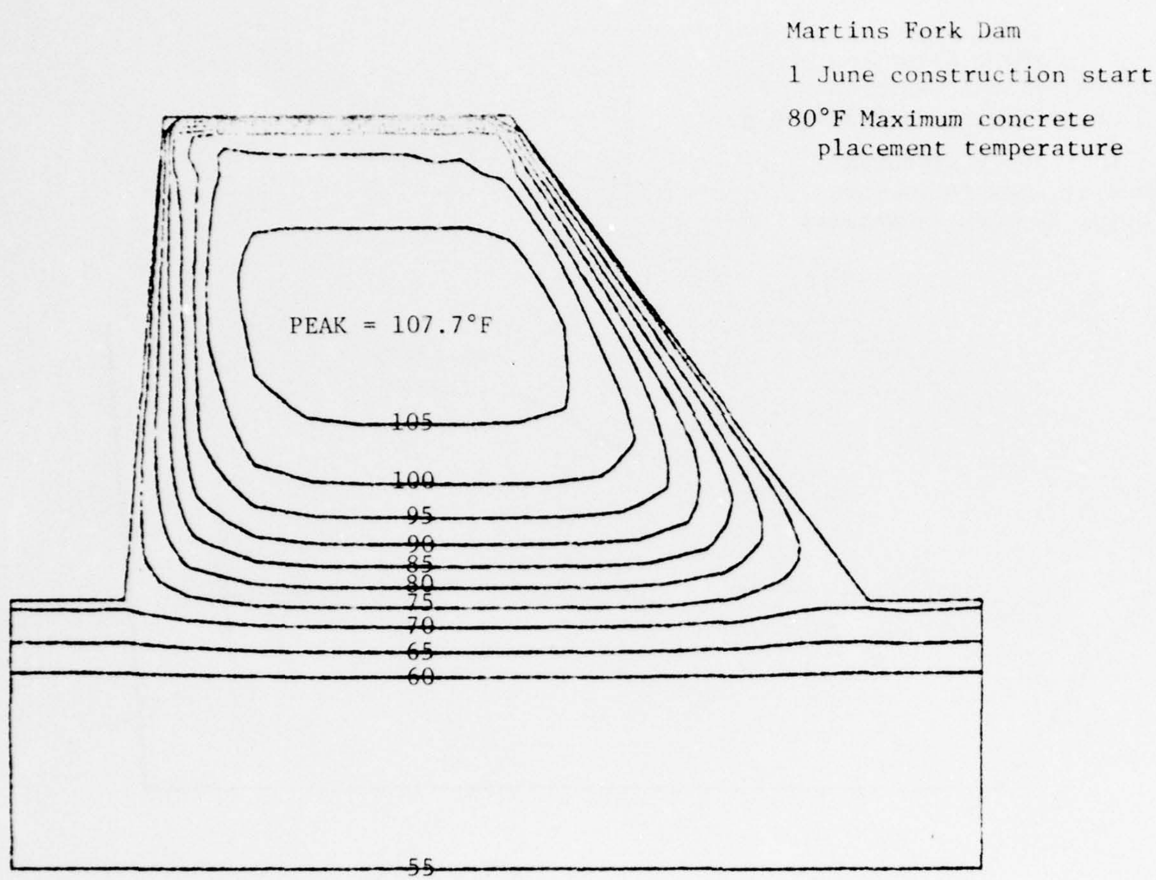
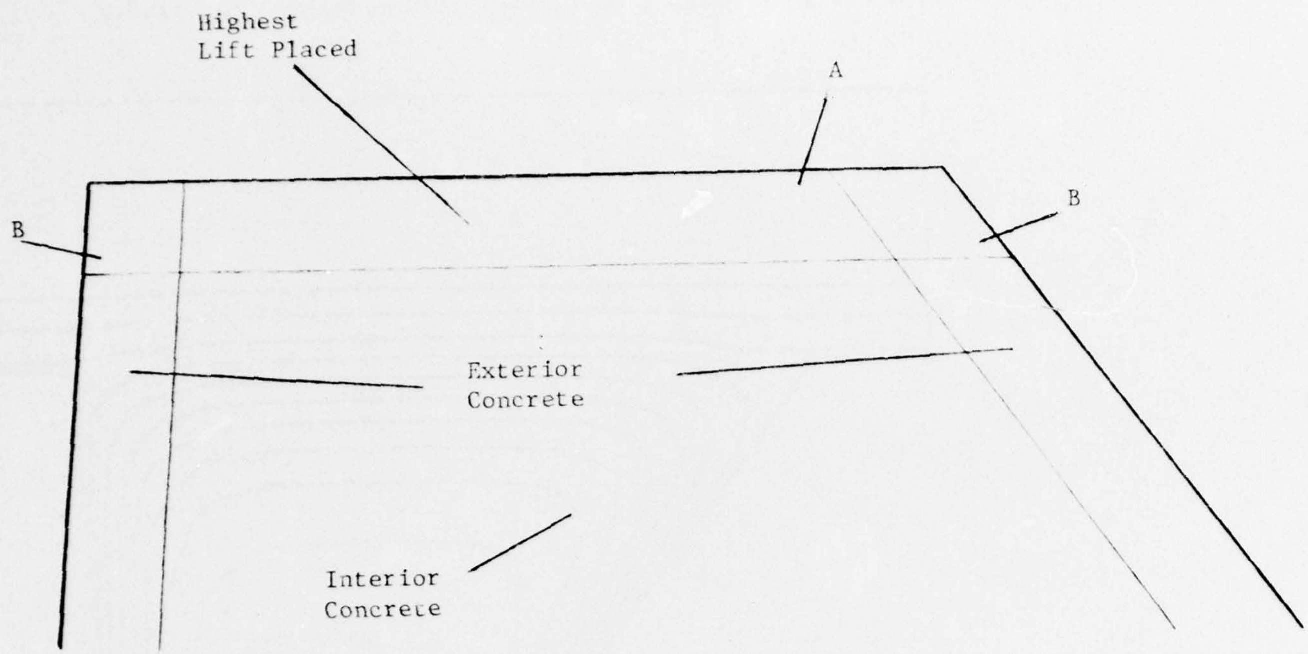


Figure 10. Isotherm plot (deg F) at 5 days after placement of Lift 14



A - Represents area at which maximum horizontal tensile strains typically occur

B - Represents areas at which maximum vertical tensile strains typically occur

Figure 11. Areas of maximum tensile strain during exposure of Lifts 4-14

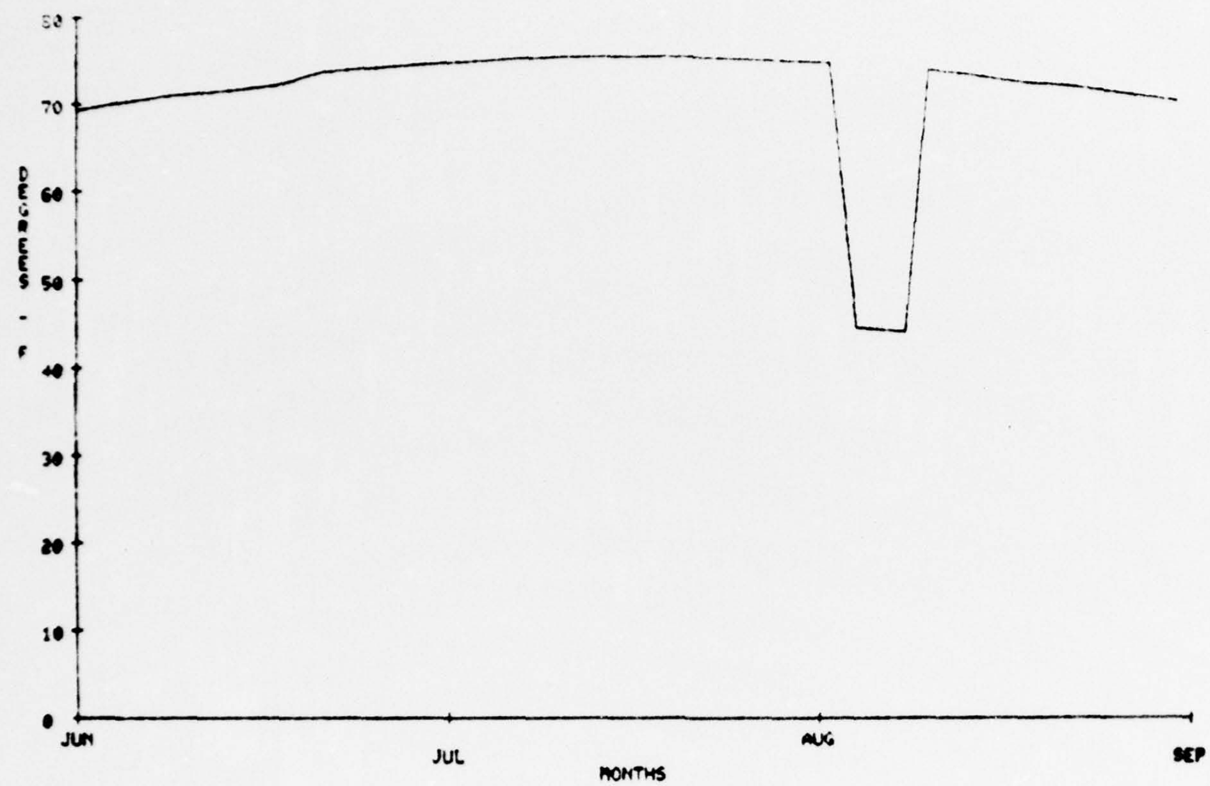


FIGURE 12 RUN 8 AMBIENT ENVIRONMENT

In accordance with ER 70-2-3, paragraph 6c(1)(b), dated 15 February 1973, a facsimile catalog card in Library of Congress format is reproduced below.

Bombich, Anthony A
Determination of maximum concrete placement temperatures for Martins Fork Dam, by Anthony A. Bombich. Vicksburg, U. S. Army Engineer Waterways Experiment Station, 1977.
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Prepared for U. S. Army Engineer District, Nashville, Nashville, Tennessee.
Includes bibliography.

1. Computer programs. 2. Concrete placing. 3. Concrete temperature. 4. Concrete thermal properties. 5. Finite element method. 6. Martins Fork Dam. I. U. S. Army Engineer District, Nashville. (Series: U. S. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper C-77-7)
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